TA705.3S39G4 1989
Also
refer
to
TG310.S39.G6/1990

File# 18835
SARATOGA CREEK

GEOTECHNICAL ENGINEERING INVESTIGATION
SPT CO. SARATOGA CREEK BRIDGE
AND TEMPORARY SHOOFLY BRIDGE
BRIDGE NOS. 37X-18, 37-SHFY
SARATOGA, CALIFORNIA

For

Bissell & Karn, Inc.
10293 Rockingham Drive, Suite 101
Sacramento, CA 95827

PSC
ASSOCIATES, INC.

GEOTECHNICAL CONSULTANTS
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November 12, 1989

Job No. 89116.40

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PSC

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GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

SPT CO. SARATOGA CREEK BRIDGE

AND TEMPORARY SHOOFLY BRIDGE

STRUCTURE NOS. 37x-18, 37-SHFY

SARATOGA, CA

SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

Based upon our field exploration and laboratory test data, it is our opinion that the project is feasible from a geotechnical engineering standpoint.

The subsurface soil conditions generally consist of slightly compact to very dense gravelly sand and sandy gravel with silt and clay layers. Groundwater was encountered in our borings at elevations 262 and 260.

The proposed main bridge structure may be supported on either Caltrans standard pre-stressed, pre-cast, concrete piles or HP 10x57 steel piles designed for a maximum vertical load of 70-tons. The proposed temporary shoofly bridge structure may be supported on timber piles designed for maximum vertical load of 20-tons. Pre-drilling is required through the subsurface gravelly sand layers for concrete and timber piles.

As requested, the geotechnical design parameters are summarized in the following table.

The text of this report should be referred to for detailed conclusions and recommendations.

SUMMARY OF GEOTECHNICAL DESIGN PARAMETERS

SEISMIC DESIGN DATA

1.	Depth to rock-like material	>80 - <150 ft.
2.	Maximum expected acceleration at bedrock or "Rock-like" material	0.64 g.
3.	Site Period	0.8 to 1.2 sec.

LATERAL SOIL FORCES

1.	Seismic soil pressure		33% increase over active and passive cases.
2.	Active case:	Design Value	34 pcf EFP (Equivalent Fluid Pressure).
		Ultimate Value	per Caltrans multiplier
3.	At Rest case:	Design Value	48 pcf EFP (Equivalent Fluid Pressure) for native soils, or per Caltrans standards for backfill materials.
		Ultimate Value	per Caltrans multiplier
4.	Passive Resistance:	Design Value	425 pcf EFP (Equivalent Fluid Pressure) for native soils, or per Caltrans standards for backfill materials.
		Maximum Value	6250 psf (Uniform Pressure).

		70-Ton Piles		20-Ton Timber Piles
		12-inch/14-inch Concrete Piles	Steel Piles	
A.	Lateral Earth Pressure on piles at ground surface with maximum deflection of 0.25 inch	16/22 kips	20 kips	
B.	Lateral Earth Pressure on piles at elevated position with maximum deflection of 0.25 inch			6 kips

PILING SUPPORT

A. SPT Co. Saratoga Creek Bridge Structure #37x-18:
12-inch and/or 14-inch pre-stressed, pre-cast concrete piles or HP 10x57 steel piles designed for a maximum load of 70-tons are considered for bridge piling support.

B. Temporary Shoofly Bridge Structure #37-SHFY:
20-ton timber piles are considered for bridge piling support.

Detail tables showing specified and probable tip elevations, pre-drilling elevations and the capacities are provided in the text of the report.

INTRODUCTION

This report presents the results of our Geotechnical Engineering Investigation for the proposed main and temporary Southern Pacific Transportation Company bridge structures over Saratoga Creek in the City of Saratoga, California. Our work was performed generally in accordance with the scope of work as outlined in our proposal dated April 18, 1989.

PURPOSE AND SCOPE

The purpose of this investigation was to evaluate the general soil conditions at the project site, to determine their engineering properties and to provide recommendations for bridge foundation support. Corrosion study of the existing subsurface soils is provided in the Materials Report of Route 85 by Woodward Clyde Consultants under Section 7.0. For convenience, we have included excerpts from that study in this report.

The scope of work performed for this investigation included a review of readily available soils and geologic literature pertaining to the site; obtaining representative samples and logging soil materials encountered in two exploratory borings; laboratory testing of the collected samples; engineering analysis of the field and laboratory data; and preparation of this report. The basis for this investigation were preliminary site plan prepared by Bissell & Karn, Inc. We redrafted the bridge alignment and profile on Plates 1 and 2, which also indicate the approximate boring locations and the log of borings at this location.

Due to limitations inherent in geotechnical investigations, it is neither uncommon to encounter unforeseen variations in the soil conditions during construction nor is it practical to determine all such variations during an acceptable program of drilling and sampling for a project of this scope. Such variations, when encountered, generally require additional engineering services to attain properly constructed project.

Our recommendations in this report are based on the above information. Any major deviation should be reported to this office for consideration.

SITE CONDITIONS

The proposed bridge site over Saratoga Creek is located approximately 850 feet north of Saratoga Avenue and proposed Route 85 in the City of Saratoga, California. The existing Southern Pacific railroad bridge alignment is approximately 40 feet to the east of the proposed bridge end. Several pipelines belonging to Santa Clara Valley Water District cross the creek to the west of the existing bridge. Saratoga Creek flows east through relatively level topography in this area. The existing creek banks are about 14 feet high at an estimated average slope gradient of 1½:1 (H:V). The creek has a flat bottom at an elevation of 304. Scattered trees and light vegetation cover most of the creek channel and banks. At the time of this investigation, shallow water was noticed in the creek channel.

PROPOSED CONSTRUCTION

A. SPT Co. Saratoga Creek Bridge:

The proposed Southern Pacific Railroad over Saratoga Creek will be a clear span structure approximately 90 feet long. The proposed bridge structure will be supported on two abutments at the original grade. The following are the proposed pile cap elevations for proposed bridge structure.

LOCATION	APPROXIMATE BOTTOM OF PILE CAP ELEVATIONS
Abutment #1	305
Abutment #2	305

The proposed channel bank slopes will be approximately 2:1 (H:V) on both sides of the creek. A future pedestrian walkway is also planned along the northern creek embankment at elevation 300.

B. Temporary Shoofly Bridge:

The proposed shoofly bridge will be a temporary bridge located northeast of the proposed main bridge. The bridge structure will be approximately 75 feet long and supported on 6 abutments and piers. The upper portion of the creek embankments will be temporarily graded during the construction of the shoofly bridge. The proposed channel banks will be at a slope gradient of 1½:1 (H:V).

FIELD EXPLORATION

Based on the plans, discussions with the client, readily available data on the area, and our site reconnaissance, two borings were drilled at the site to a maximum depth of 70 feet. The locations and description of the materials are shown on Plate 1, Log of Test Borings. Weighing structural design considerations against costs of further explorations, we believe that adequate soils information has been obtained in order to provide foundation recommendations.

The test borings were advanced with a truck mounted drill rig using eight-inch diameter drilling bits. Selected drive samples of 1.4 and 2.42 inches I.D. (Modified California Sampler) were obtained from the borings at various depths. The samples were driven into subsurface soils under the impact of a 140-pound hammer having a free fall of 30 inches. The blow counts are presented on the boring logs. (When correlating standard penetration data, the blow counts for the Modified California Sampler can be taken as roughly twice that for the Standard Penetration Test in similar soils).

The samples were sealed and transported to our laboratory for further evaluation and testing. Laboratory test results are attached in Appendix A of this report.

SOIL CONDITIONS

The subsurface soil conditions consist of slightly compact to very dense gravelly sand and sandy gravel with silt and clay layers; these layers extend to 70 feet, the maximum depth explored.

Groundwater was encountered at elevations 262 and 260. The elevation of water table may change with the passage of time due to groundwater fluctuations from season to season.

Detailed descriptions of the materials encountered in each of the exploratory borings are presented on Plate 1, "Logs of Borings". It should be noted that these descriptions and related information depict subsurface conditions only at the locations indicated on the Site Plan and on the particular date noted on the logs. Because of the variability from place to place within soils in general, subsurface conditions at other locations may differ from conditions occurring at the locations explored. Also, the passage of time may result in a change in the soil conditions at these locations due to environmental changes.

The field study was conducted under the supervision of our field engineer who logged the test borings and prepared the samples for subsequent laboratory testing and evaluation.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the findings of our investigation, it is our opinion that the site is feasible for the proposed bridge structures provided the recommendations presented in this report are incorporated into the final design and construction. The foundation type considered feasible to support the main bridge structure are Caltrans standard 70-ton pre-stressed, pre-cast concrete piles or HP 10x57 steel piles. The temporary shoofly bridge structure will be supported on 20-ton timber piles.

This report was prepared specifically for the proposed bridge locations shown on Plates 1 and 2. Normal construction procedures were assumed throughout our analysis and represent one of the basis of recommendations presented herein. Our design criteria have been based upon using the existing materials on the site. Therefore, we should be notified in the event that these conditions are changed, so as to modify or amend our recommendations.

Geology

General geologic features pertaining to the site were evaluated by reference to the State of California Division of Mines and Geology Special Report 107 (1974). Reference to this publication indicates that the site is underlain by Quaternary Younger Alluvial Fan deposits (Qyf). Depth to "rock like" material (seismic wave velocity > 2500 fps) is given as greater than 80 and less than 150 feet. Depth to first encountered groundwater is given as 35'. The site is classified as an "E-1" seismic zone (moderate potential for liquefaction, lurch cracking, and lateral spreading).

Seismicity

The San Francisco Bay area is recognized by geologists and seismologists as one of the most active seismic regions in the United States. The significant earthquakes which occur in this area are generally associated with crustal movements along well-defined active fault zones. The proposed project site does not lie within an Alquist-Priolo Special Fault Study Zone. No mapped faults are shown trending through or toward the project site. Fault data for nearby mapped faults are presented in the following table:

FAULT	DISTANCE (MILES) DIRECTION	MAXIMUM CREDIBLE EARTHQUAKE MAGNITUDE (1)	ACTIVE (A) POTENTIALLY ACTIVE (PA) (2)	PEAK BEDROCK ACCELERATION (3) (g)
San Andreas	4.3 SW	8.25	A	0.64
Hayward	13.8 NE	7.5	A	0.35
Calaveras	15.0 NE	7.5	A	0.35
Evergreen	12.7 NE	6.5	PA	0.25
Silver Creek	11.0 NE	6.5	PA	0.28
Shannon	0.1 SW	6.5	PA	0.6

(1) Greensfelder, 1972

(2) CDMG SR-107, 1974

(3) Schnable and Seed, 1973

The controlling fault for design is the San Andreas. Potential seismic hazards at the site may arise from three sources: faulting, shaking and liquefaction. Since no mapped fault appears to pass through the site, the potential risk from fault offset is, in our opinion, considered low. However, on the basis of available information, it is prudent to assume the site would be subject to severe ground shaking caused by a major earthquake during the life of the structure.

The soils that are most susceptible to liquefaction are loose, poorly graded, clean fine sands, and non-cohesive silts. In addition, these materials must be below the water table (or at least saturated) for this phenomenon to occur. As described above under "Soil Conditions" and shown on the boring logs, loose saturated sands and silts were not encountered.

We, therefore, believe that the potential for secondary ground failure due to subsurface liquefaction is low.

Drainage

Because soils, in general, tend to lose strength when they become wet, it is essential that drainage be properly controlled. Grading with engineered fill, if any, should be performed so as to provide positive drainage away from proposed slopes.

Foundations

A. Main Bridge Structure #37x-18

The proposed bridge structure should be supported on Caltrans standard pre-stressed, pre-cast concrete pile or steel H-pile foundation system.

Based on the subsurface conditions and CalTrans specifications, the pile size considered here are 12-inch and/or 14-inch prestressed, precast concrete or HP 10x57 steel piles. Both types of piles are designed for a maximum vertical load of 70-tons.

B. Temporary Shoofly Bridge Structure #37-SHFY

As requested, the proposed temporary bridge can be supported on timber pile foundation system.

Based on the subsurface conditions, the timber piles considered here should be capable to support maximum vertical load of 20-tons.

A minimum factor of safety of 2.0 for skin resistance and 3.0 for end bearing capacities have been used to obtain allowable values.

Pre-drilling through the fill embankment should be accomplished in accordance with Caltrans specifications. Pre-drilling through the upper native subsurface layers should be accomplished with continuous flight auger of diameter not to exceed 70 percent of the pile's smallest dimension. Piles in groups should have a minimum center-to-center spacing of 36-inches. The following are specified and the probable elevations, and pre-drilling elevations.

A. SPT Co. Saratoga Creek Bridge Structure #37X-18

12-INCH & 14-INCH CONCRETE PILES
(Caltrans Standard 70-Ton)

Location	Specified Tip Elevation	Probable Tip Elevation	Pre-drilling Elevation
Abutment #1	267	257	262
Abutment #2	261	256	266

HP 10X57 STEEL PILES

Location	Specified Tip Elevation	Probable Tip Elevation
Abutment #1	255	250
Abutment #2	254	249

B. Temporary Bridge Structure #37-SHFY

20-TON TIMBER PILES

Location	Specified Tip Elevation	Probable Tip Elevation	Pre-drilling Elevation
Abutment #1	273	268	278
Pier #2	268	263	273
Pier #3	268	263	273
Pier #4	268	263	273
Pier #5	272	267	277
Abutment #6	279	274	284

All piles should have bearing in accordance with the Engineering News Formula at final tip elevation. If the minimum should penetrate at least to the tip elevation, unless otherwise permitted in writing by the engineer. If the minimum specified tip elevation is reached without achieving bearing, pile driving should continue until bearing is obtained.

Pile hammers, other than drop hammers, should be approved steam, air, or diesel hammers that develop sufficient energy to drive the piles at penetration rate of not less than 1/8 inch per blow at the required bearing value.

We recommend that our firm be provided the opportunity to review the final design and specifications to see that the pile foundation recommendations are properly interpreted and implemented in the design and specifications.

Lateral Design for Piles

The maximum allowable lateral loads for 12-inch, 14-inch concrete, and steel piles are 16, 22, and 20 kips, respectively, these loads are applicable for rigid cap at the ground surface. The maximum allowable lateral loads for 20-ton timber piles is 6 kips which is applicable for the shoofly bridge structure at elevated position. All loads are calculated for maximum deflection of one-quarter inch at the ground surface. If greater deflection can be tolerated, lateral loads can be increased directly in proportion to the deflection up to twice the recommended loads. It is estimated that the point of fixity to be 8 feet below the ground surface.

Passive earth pressure of undisturbed or compacted native soils may be compacted as an equivalent fluid having a density of 425 pounds per cubic feet and maximum passive earth pressure of 6250 pounds per square foot for one inch movement at seismic condition.

For lateral resistance to sliding, a 0.35 coefficient of friction may be considered. In the event passive earth pressure and friction are considered for lateral resistance, the passive component should be reduced by one-third for transient or seismic loads.

Caltrans Seismic Design Criteria

For Caltrans seismic design criteria, we recommend a peak bedrock acceleration of 0.64 g. The depth of soil to rocklike material (seismic wave velocity >2500 fps) should be considered greater than 80 and less than 150 feet. For seismic design considerations, the allowable pile capacities provided in previous tables may be increased by 100% to obtain the ultimate capacities. Based on the equivalent single layer method proposed by Seed, we estimate the characteristic site period to be within the range of 0.8 to 1.2 seconds.

Retaining Walls, Wingwalls

The retaining walls should be designed to resist the applied lateral earth pressures presented in the following table. These values are for unrestrained (cantilever) conditions. These values assume no hydrostatic pore pressure buildup behind the wall and are based on well drained backfill behind the walls.

Applied Lateral Earth Pressure
(EFP)

Design Conditions	Level Backfill	2:1 Backfill Slope
(A) Native Soils		
Active	34 pcf	46 pcf
At-Rest	48 pcf	65 pcf

(B) Compacted Fill: As per Caltrans Specifications

Cantilever walls which are free to rotate at least 0.001 radian may be assumed flexible and designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) should be added to the preceding lateral earth pressures. A coefficient of 0.3 may be used to determine the additional earth pressure resulting from the surcharge.

To account for seismic effects on the walls, we recommend that an additional 33 percent of equivalent fluid pressure be used for design of the walls. The application of the resultant force should be located similarly as for static loading conditions. The estimated angle of internal friction and unit weight of the upper gravelly sand soils are 38 degree and 130 pound per cubic foot, respectively. Temporary shoring should be designed in accordance with Caltrans Division of Structure's Trenching and Shoring Manual.

Corrosion Study

The detailed Materials Report prepared for the Santa Clara County Traffic Authority by Woodward Clyde Consultants addresses the corrosion study for the project. Our scope of work did not require additional testing and reporting. Following is an excerpt from the Materials Report Section 7.0.

7.47 Saratoga Avenue Crossing (Station 411+50)

> "The soils tested at the proposed bridge and undercrossing sites is fairly to mildly corrosive to buried metallic structures. However, the slightly alkaline pH and very low chloride concentrations make the potential for deterioration of concrete structures in this environment to be negligible."
>
> "Assuming a worst case chloride concentration of 200 ppm (forty times the maximum concentration measured at this location) the estimated time to corrosion of the bridge substructures is 50 years or more (for 6 sack mix, 2 inch cover reinforcing steel and 15 percent total mix water) as per California Test 532, Figure 2."

Plan Review

We recommend that final plans for foundations be reviewed by this office prior to construction so that the intent of our recommendations is included in the project plans and specifications and to further see that no misunderstandings or misinterpretations have occurred.

Construction Observation

To a degree, the performance of any structure is dependent upon construction procedures and quality. Hence, density testing of subgrades and observation of foundation excavations and pile driving should be carried out by the regulating agencies. Any subsurface conditions different from those forming the basis of our recommendations should be forwarded to assess the need for design changes. Therefore, the recommendations presented in this report are contingent upon good quality control and these geotechnical observations during construction.

INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted geotechnical engineering principles and practices and are based on our site reconnaissance and the assumption that the soil conditions do not deviate from observed conditions. All work done is in accordance with generally accepted geotechnical engineering principles and practices. No warranty, expressed or implied, of merchantability or fitness, is made or intended in connection with our work or by the furnishing of oral or written reports or findings. Unanticipated soil conditions are commonly encountered and cannot be fully determined by taking soil samples and excavating test borings; different soil conditions may require that additional expenditures be made during construction to attain a properly constructed project. Some contingency fund is thus recommended to accommodate these possible extra costs.

This report has been prepared for the proposed bridges as described earlier, to assist the engineer in the design of this project. In the event any changes in the design or location of the facilities are planned, or if any variations or undesirable conditions are encountered during construction, our conclusions and recommendations shall not be considered valid unless the changes or variations are reviewed and our recommendations modified or approved by us in writing.

This report is issued with the understanding that it is the designer's responsibility to ensure that the information and recommendations contained herein are incorporated into the project and that necessary steps are also taken to see that the recommendations are carried out in the field.

The findings in this report are valid as of the present date. However, changes in the soil conditions can occur with the passage of time, whether they be due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or from the broadening of knowledge. Accordingly, the findings in this report might be invalidated, wholly or partially, by changes outside of our control.

Therefore, this report is subject to review by the controlling government agencies and is valid for a period of one year.

Respectfully submitted,

P S C ASSOCIATES, INC.

John W. Farage
Project Engineer

Gary Parikh
G.E. 666

JWF/GP:lj

APPENDIX A

PLATE 1

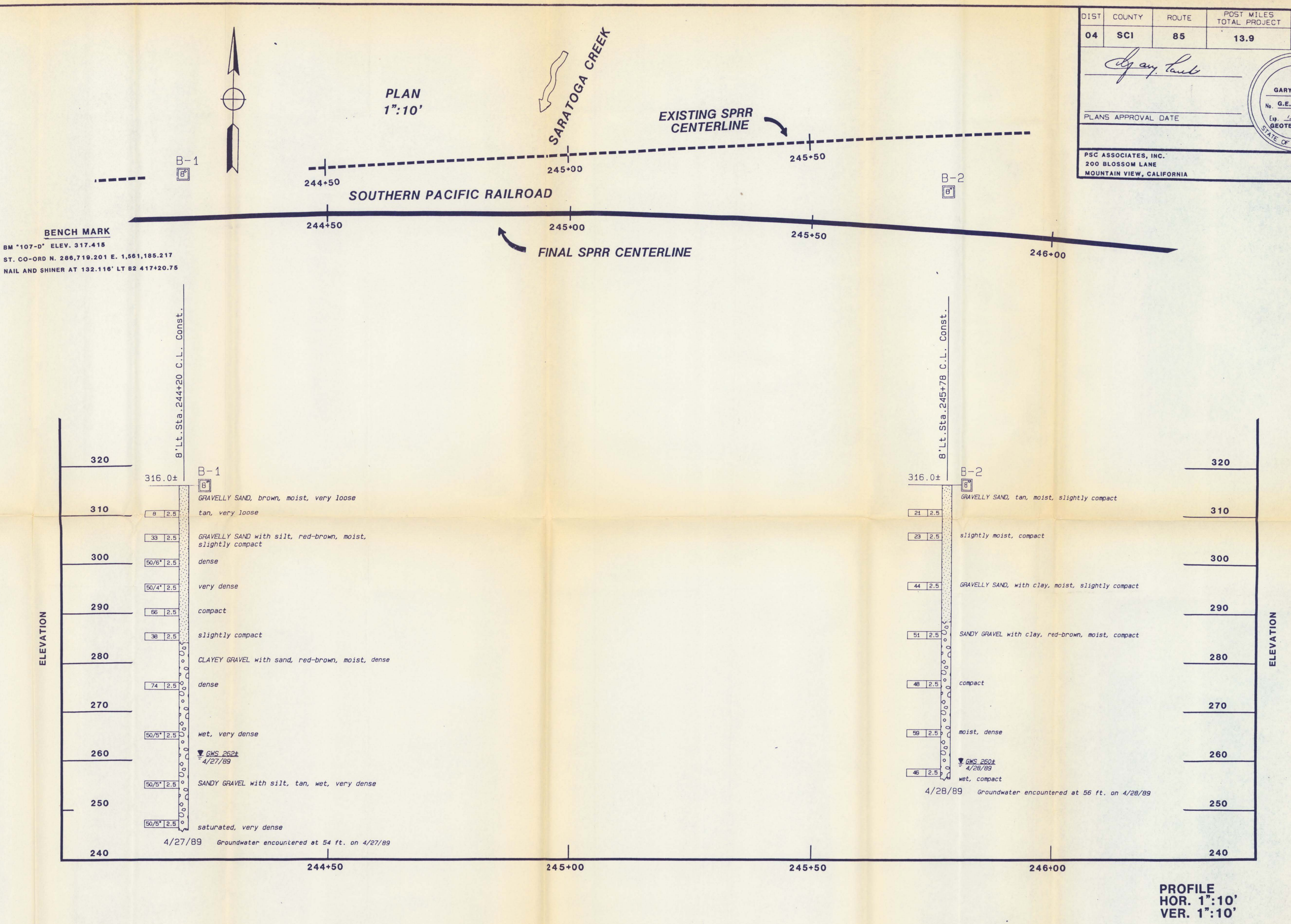
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SCI	85	13.9	1	2

Gary Parikh

PLANS APPROVAL DATE

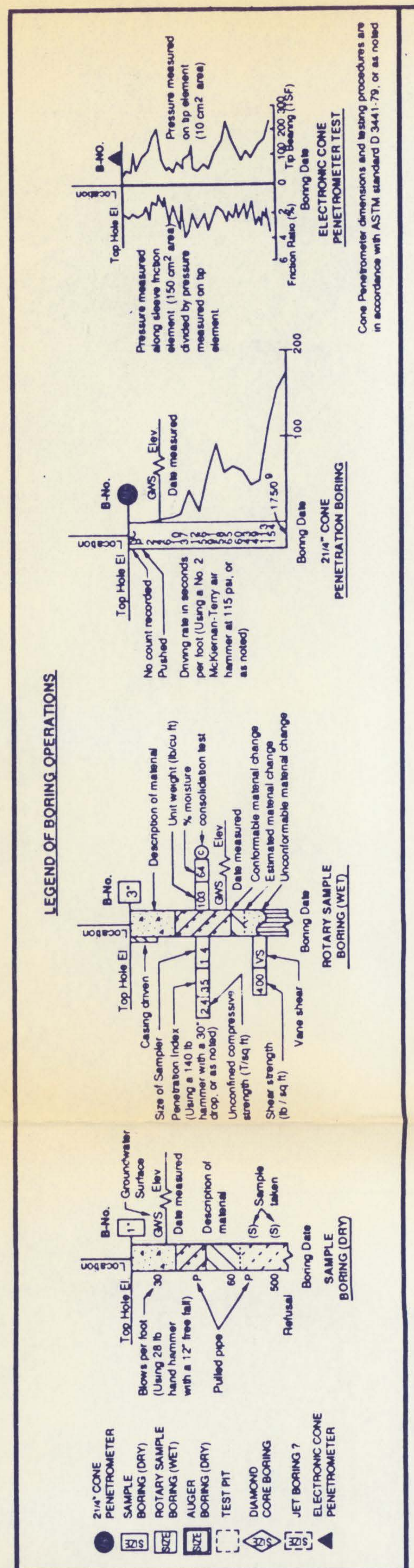
GARY PARIKH
No. G.E. 666
Exp. 12/31/1989
GEOTECHNICAL
STATE OF CALIFORNIA

PSC ASSOCIATES, INC.
200 BLOSSOM LANE
MOUNTAIN VIEW, CALIFORNIA


PLAN
1":10'
SARATOGA CREEK
EXISTING SPRR CENTERLINE
B-1
8"
244+50
245+00
245+50
B-2
8"
SOUTHERN PACIFIC RAILROAD
244+50
245+00
245+50
246+00
FINAL SPRR CENTERLINE
BENCH MARK
BM "107-D" ELEV. 317.415
ST. CO-ORD N. 286,719.201 E. 1,561,185.217
NAIL AND SHINER AT 132.116' LT B2 417+20.75
ELEVATION
320
310
300
290
280
270
260
250
240
8'Lt.Sta.244+20 C.L. Const.
316.0±
B-1
8"
8 2.5
33 2.5
50/6" 2.5
50/4" 2.5
66 2.5
38 2.5
74 2.5
50/5" 2.5
50/5" 2.5
50/5" 2.5
GRAVELLY SAND, brown, moist, very loose
tan, very loose
GRAVELLY SAND with silt, red-brown, moist, slightly compact
dense
very dense
compact
slightly compact
CLAYEY GRAVEL with sand, red-brown, moist, dense
dense
wet, very dense
GWS 262±
4/27/89
SANDY GRAVEL with silt, tan, wet, very dense
saturated, very dense
4/27/89 Groundwater encountered at 54 ft. on 4/27/89
8'Lt.Sta.245+78 C.L. Const.
316.0±
B-2
8"
21 2.5
23 2.5
44 2.5
51 2.5
48 2.5
59 2.5
46 2.5
GRAVELLY SAND, tan, moist, slightly compact
slightly moist, compact
GRAVELLY SAND, with clay, moist, slightly compact
SANDY GRAVEL with clay, red-brown, moist, compact
compact
moist, dense
GWS 260±
4/28/89
wet, compact
4/28/89 Groundwater encountered at 56 ft. on 4/28/89
244+50
245+00
245+50
246+00
ELEVATION
320
310
300
290
280
270
260
250
240

PROFILE
HOR. 1":10'
VER. 1":10'

PSC PROJECT #89116.40


LEGEND OF BORING OPERATIONS
2 1/4" CONE PENETROMETER
SAMPLE BORING (DRY)
ROTARY SAMPLE BORING (WET)
AUGER BORING (DRY)
TEST PIT
DIAMOND CORE BORING
JET BORING ?
ELECTRONIC CONE PENETROMETER
SIZE
Top Hole El
Location
B-No.
1"
Groundwater Surface
GWS
Elev
Date measured
Blows per foot (Using 28 lb hand hammer with a 12" free fall)
30
60
500
P
Pulled pipe
Description of material
(S) Sample taken
Refusal
Boring Date
SAMPLE BORING (DRY)
Top Hole El
Location
B-No.
3"
Casing driven
Size of Sampler
Penetration Index (Using a 140 lb hammer with a 30" drop, or as noted)
Unconfined compressive strength (T/sq ft)
Shear strength (lb / sq ft)
Vane shear
2.4 35 1.4
103 64 C
400 VS
Description of material
Unit weight (lb/cu ft)
% moisture
consolidation test
GWS Elev
Date measured
Conformable material change
Estimated material change
Unconformable material change
Boring Date
ROTARY SAMPLE BORING (WET)
Top Hole El
Location
B-No.
No count recorded
Pushed
Driving rate in seconds per foot (Using a No 2 McKiernan-Terry air hammer at 115 psi, or as noted)
GWS Elev
Date measured
1750
100
200
Boring Date
2 1/4" CONE PENETRATION BORING
Top Hole El
Location
B-NO.
Pressure measured along sleeve friction element (150 cm² area) divided by pressure measured on tip element
Pressure measured on tip element (10 cm2 area)
6 4 2 0
Friction Ratio (%)
100 200 300
Tip Bearing (TSF)
Boring Date
ELECTRONIC CONE PENETROMETER TEST
Cone Penetrometer dimensions and testing procedures are in accordance with ASTM standard D 3441-79, or as noted

LEGEND OF EARTH MATERIALS

- GRAVEL
- SAND
- SILT
- CLAY
- SANDY CLAY or CLAYEY SAND
- SANDY SILT or SILTY SAND
- SILTY CLAY
- CLAYEY SILT
- PEAT and/or ORGANIC MATTER
- FILL MATERIAL
- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

Penetration Index (Blows / Ft)	Granular	Cohesive
0-4	Very loose	Very soft
5-9	Loose	Soft
10-19	Slightly compact	Stiff
20-34	Compact	Very stiff
35-69	Dense	Hard
>70	Very dense	Very hard

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.

CALTRANS STRUCTURES PROJECT REVIEWER

REGISTERED CIVIL ENGINEER NO. ____

DRAWN BY	B.M. DZUGAJ	J. W. Farage FIELD INVESTIGATOR	
CHECKED BY	J. W. FARAGE	Date 6/20/1989	

PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

J. W. Farage
PROJECT ENGINEER

BRIDGE NO.	
37X-13	SPT Co. SARATOGA CREEK BRIDGE
POST MILE	
13.9	LOG OF TEST BORINGS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 04285
EA 437521

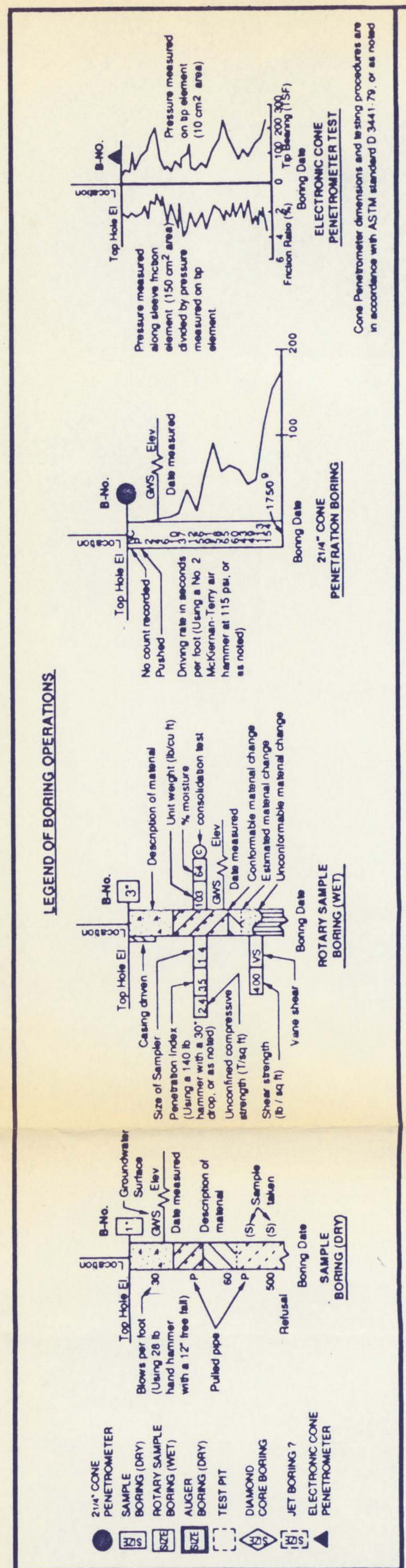
DISREGARD PRINTS BEARING EARLIER REVISION DATES →

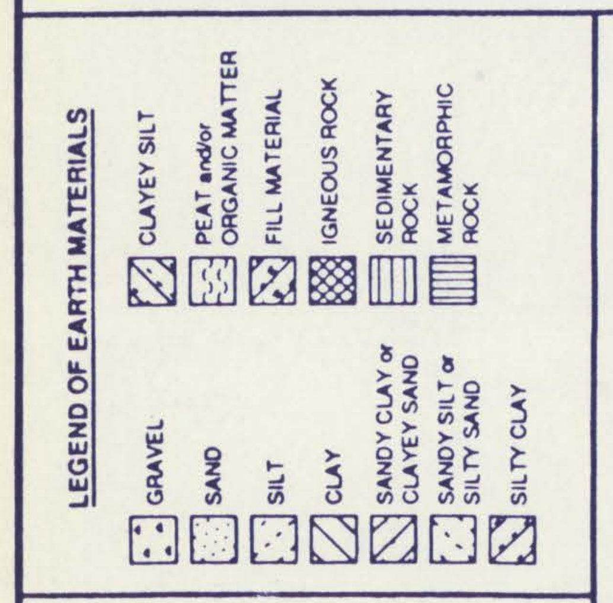
REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET	OF
1	2

PLATE 1

PLATE 2


LEGEND OF BORING OPERATIONS
Top Hole El
Location
B-No.
Groundwater Surface
GWS
Elev
Date measured
Description of material
Blows per foot (Using 28 lb hand hammer with a 12" free fall)
30
P
60
P
500
Pulled pipe
Refusal
(S)
(S)
Sample taken
Boring Date
SAMPLE BORING (DRY)
1"
3"
Casing driven
Size of Sampler
Penetration Index (Using a 140 lb hammer with a 30" drop, or as noted)
Unconfined compressive strength (T/sq ft)
Shear strength (lb / sq ft)
Vane shear
2.4
35
1.4
400
VS
103
64
C
Unit weight (lb/cu ft)
% moisture
consolidation test
Conformable material change
Estimated material change
Unconformable material change
ROTARY SAMPLE BORING (WET)
No count recorded
Pushed
Driving rate in seconds per foot (Using a No 2 McKiernan-Terry air hammer at 115 psi, or as noted)
1750
100
200
21/4" CONE PENETRATION BORING
Pressure measured along sleeve friction element (150 cm² area) divided by pressure measured on tip element
Pressure measured on tip element (10 cm2 area)
6
4
2
0
100
200
300
Friction Ratio (%)
Tip Bearing (TSF)
ELECTRONIC CONE PENETROMETER TEST
Cone Penetrometer dimensions and testing procedures are in accordance with ASTM standard D 3441-79, or as noted
● 21/4" CONE PENETROMETER
SIZE SAMPLE BORING (DRY)
SIZE ROTARY SAMPLE BORING (WET)
SIZE AUGER BORING (DRY)
TEST PIT
SIZE DIAMOND CORE BORING
SIZE JET BORING ?
▲ ELECTRONIC CONE PENETROMETER

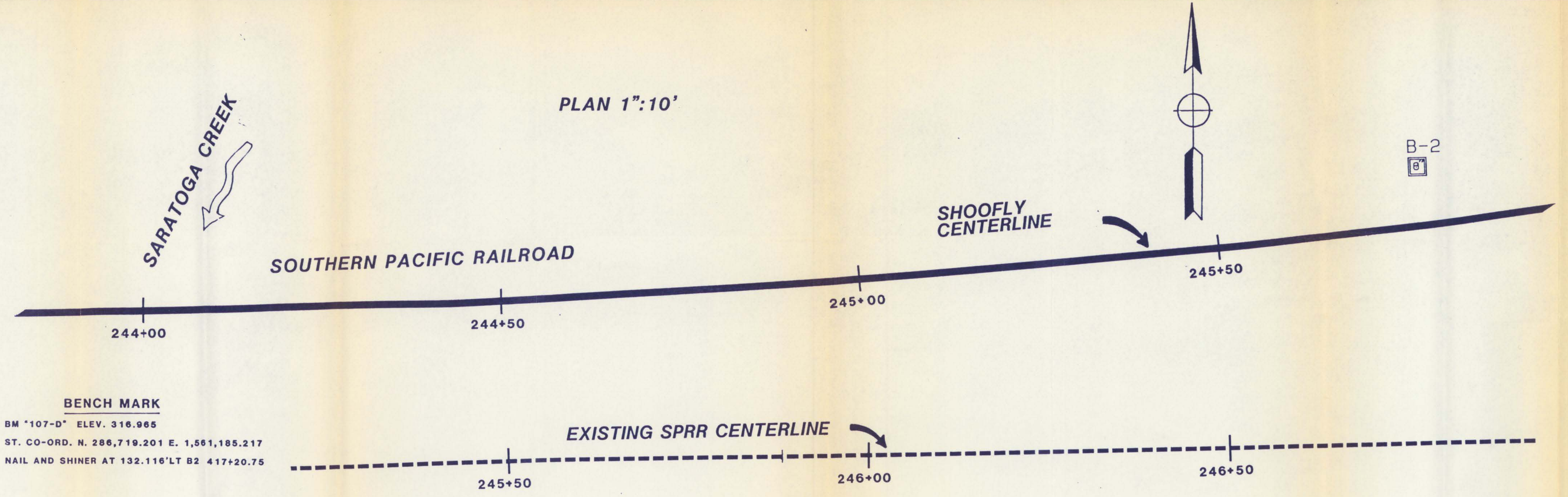


CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

Penetration Index (Blows / Ft)	Granular	Cohesive
0-4	Very loose	Very soft
5-9	Loose	Soft
10-19	Slightly compact	Stiff
20-34	Compact	Very stiff
35-69	Dense	Hard
>70	Very dense	Very hard

NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis.



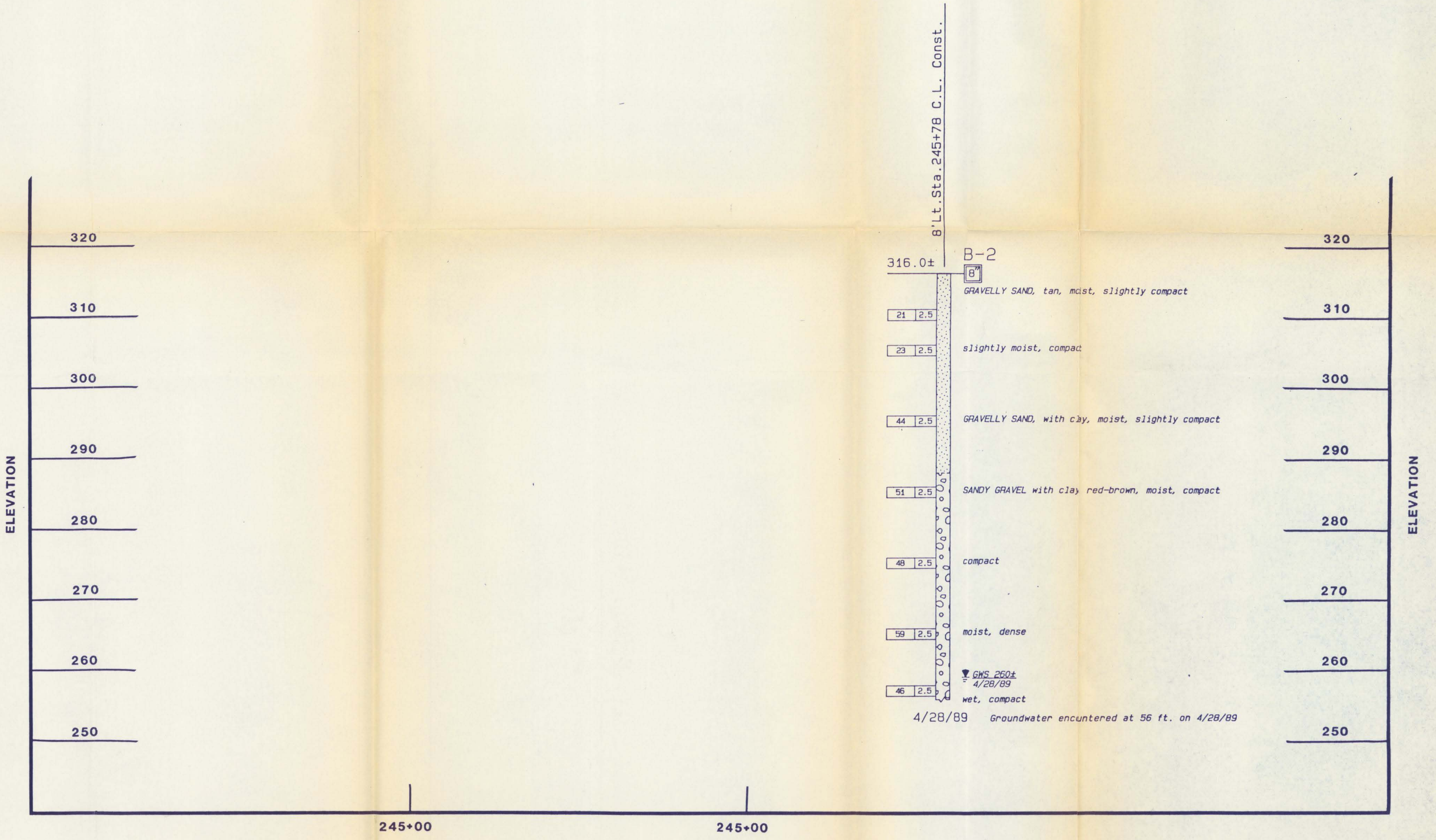
BENCH MARK

BM "107-D" ELEV. 316.965

ST. CO-ORD. N. 286,719.201 E. 1,561,185.217

NAIL AND SHINER AT 132.116'LT B2 417+20.75

PROFILE
HOR. 1":10'
VER. 1":10'


ELEVATION
320
310
300
290
280
270
260
250
8'Lt.Sta.245+78 C.L. Const.
316.0±
B-2
8"
21 2.5
23 2.5
44 2.5
51 2.5
48 2.5
59 2.5
46 2.5
GRAVELLY SAND, tan, moist, slightly compact
slightly moist, compact
GRAVELLY SAND, with clay, moist, slightly compact
SANDY GRAVEL with clay red-brown, moist, compact
compact
moist, dense
GWS 260±
4/28/89
wet, compact
4/28/89
Groundwater encountered at 56 ft. on 4/28/89
ELEVATION
320
310
300
290
280
270
260
250
245+00
245+00

DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS
04	SCI	85	13.9	2	2

Gary Parikh

PLANS APPROVAL DATE

GARY PARIKH
No. G.E. 666
Exp. 12/31/1989
GEOTECHNICAL

PSC ASSOCIATES, INC.
200 BLOSSOM LANE
MOUNTAIN VIEW, CALIFORNIA

PSC PROJECT #89116.40

CALTRANS STRUCTURES PROJECT REVIEWER	DRAWN BY	B. M. DZUGAJ	J. W. Farage FIELD INVESTIGATOR		PREPARED FOR THE STATE OF CALIFORNIA DEPARTMENT OF TRANSPORTATION
REGISTERED CIVIL ENGINEER NO.	CHECKED BY	J. W. FARAGE	Date 11/10/1989		

J. W. Farage
PROJECT ENGINEER

BRIDGE NO.
37-SHFY
POST MILE
13.9

SPT Co. SARATOGA CREEK (SHOOFLY)
LOG OF TEST BORINGS

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3

CU 04285
EA 437521

DISREGARD PRINTS BEARING EARLIER REVISION DATES →

REVISION DATES (PRELIMINARY STAGE ONLY)

SHEET 2 OF 2

PLATE 2

LABORATORY TESTS

Classification Tests

The field classification of the samples was visually verified in the laboratory in accordance with the Unified Soil Classification System. The results are presented on the "Logs of Borings", Plate 1.

Moisture-Density

The natural moisture contents and dry unit weights were determined for selected undisturbed samples of the soils. This information was used to classify and correlate the soils. The results are presented at the appropriate depths on the Boring Logs and on the "Summary of Laboratory Tests Results", Plates 3 and 4.

Gradation Analysis

Gradation tests were performed on representative samples to aid in the classification of the soil. The results are presented on Plates 7 through 9.

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SOILS, FOUNDATION AND GEOLOGICAL ENGINEERS

SPT Co. Saratoga Creek Bridge
Structures

DATE : 9/89

JOB NO : 89116.40

PLATE 3

SAMPLE NO.	U.S.C.S.*	GRAIN SIZE ANALYSIS				ATTERBERG LIMITS			STRENGTH PARAMETERS		MOIST. CONT. %	DRY DENSITY P.C.F.	REMARKS
		GRAVEL %	SAND %	SILT %	CLAY %	LIQUID %	PLASTIC %	P.I.	COH. Ksf	Ø DEGREES			
1-1	SW										7.4	-	
1-2	SW								0.12	48	-	-	See Direct Shear Diagram Plate #5
1-3	GW	66	33	1							4.1	-	See Grain Size Analysis Plate #7
1-4	GW										6.2	117	
1-5	GW										6.6	124	
1-6	GW										6.2	80	
1-7	GC										10.1	128	
1-8	GC										8.7	-	
1-9	GW	63	30	7							7.2	-	See Grain Size Analysis Plate #8
1-10	GW										11.0	-	
2-1	SW										4.4	-	
2-2	SW										3.0	107	
2-3	SW										6.5	-	
2-4	GW								0.5	31	9.5	99	See Direct Shear Diagram Plate #6
2-5	GW										6.2	114	

*UNIFIED SOIL CLASSIFICATION SYSTEM

SUMMARY OF LABORATORY TEST RESULTS

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Structures

DATE : 9/89

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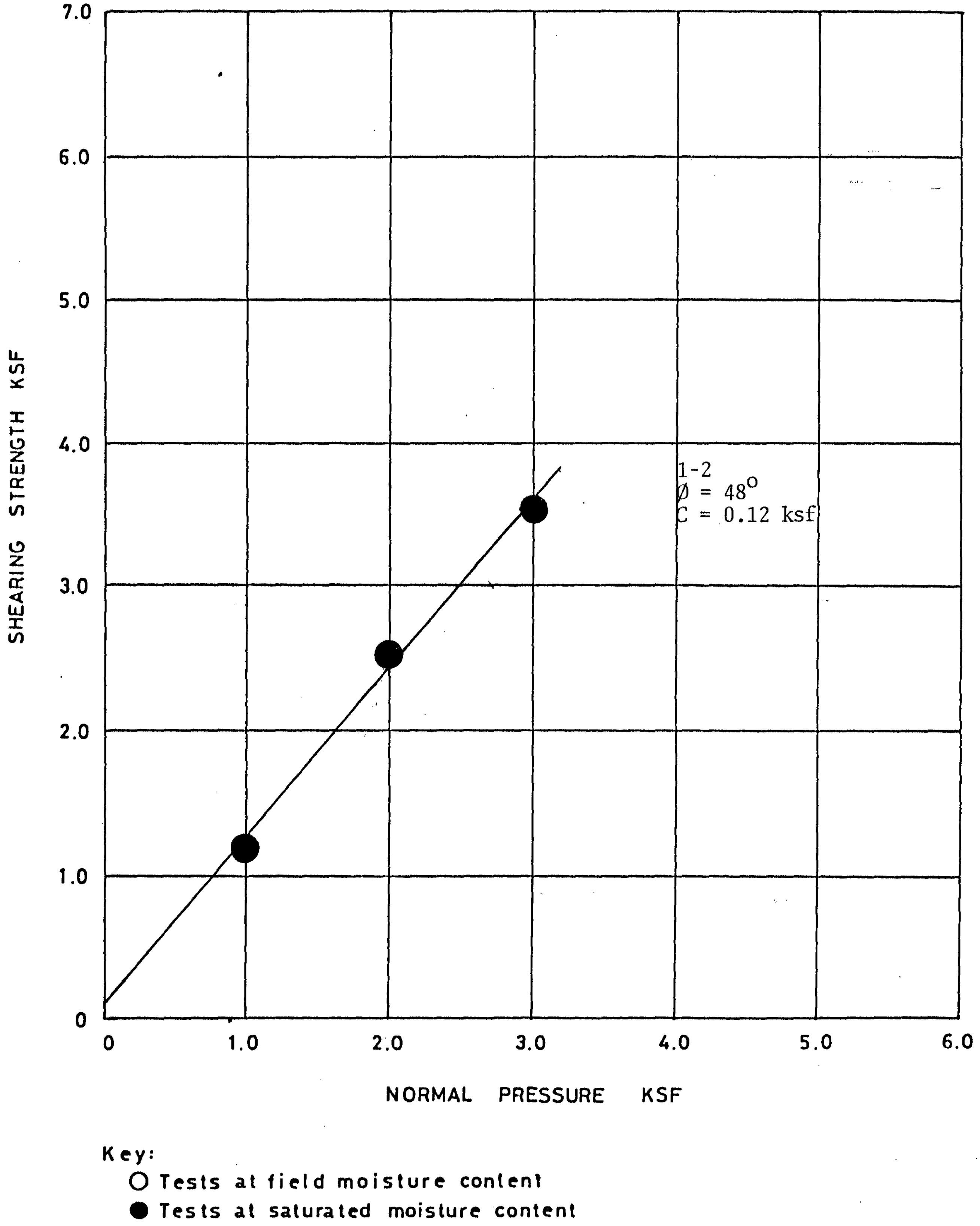
SAMPLE NO.	U.S.C.S.*	GRAIN SIZE ANALYSIS				ATTERBERG LIMITS			STRENGTH PARAMETERS		MOIST. CONT. %	DRY DENSITY P.C.F.	REMARKS
		GRAVEL %	SAND %	SILT %	CLAY %	LIQUID %	PLASTIC %	P.I.	COH. Ksf	Ø DEGREES			
2-6	GW	47	39								6.9	-	See Grain Size Analysis Plate #9
2-7	GW										16.5	119	

* UNIFIED SOIL CLASSIFICATION SYSTEM

SUMMARY OF LABORATORY TEST RESULTS

SHEAR TEST DIAGRAM



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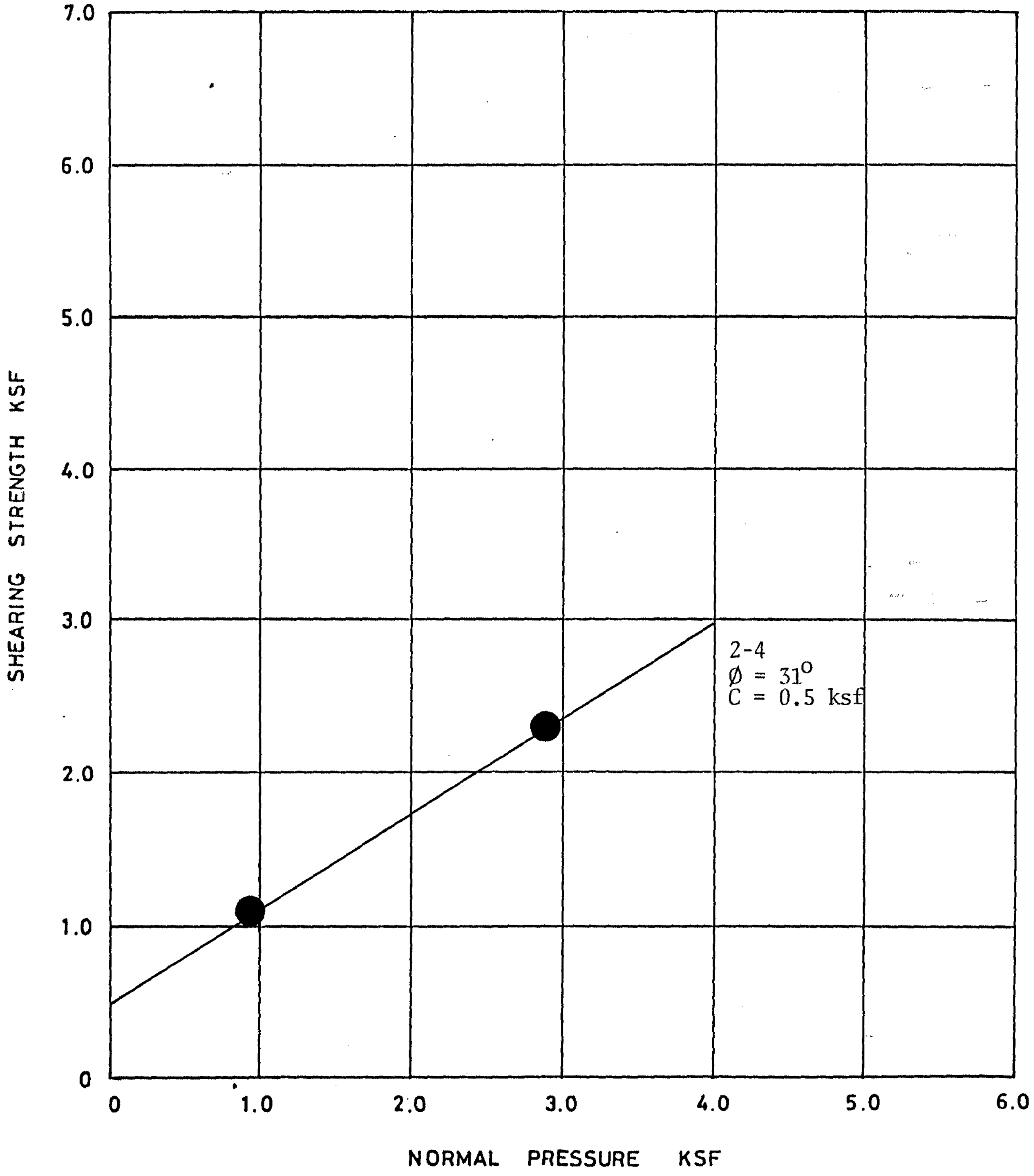
SOILS, FOUNDATION AND GEOLOGICAL ENGINEERS

SPT Co. Saratoga Creek Bridge Structures

DATE : 9/89

JOB NO : 89116.40

SHEAR TEST DIAGRAM



Key:

○ Tests at field moisture content

● Tests at saturated moisture content

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SOILS, FOUNDATION AND GEOLOGICAL ENGINEERS

SPT Co. Saratoga Creek Bridge Structures

DATE : 9/89

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Geotechnical Consultants
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Mountain View, CA 94041
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Structures

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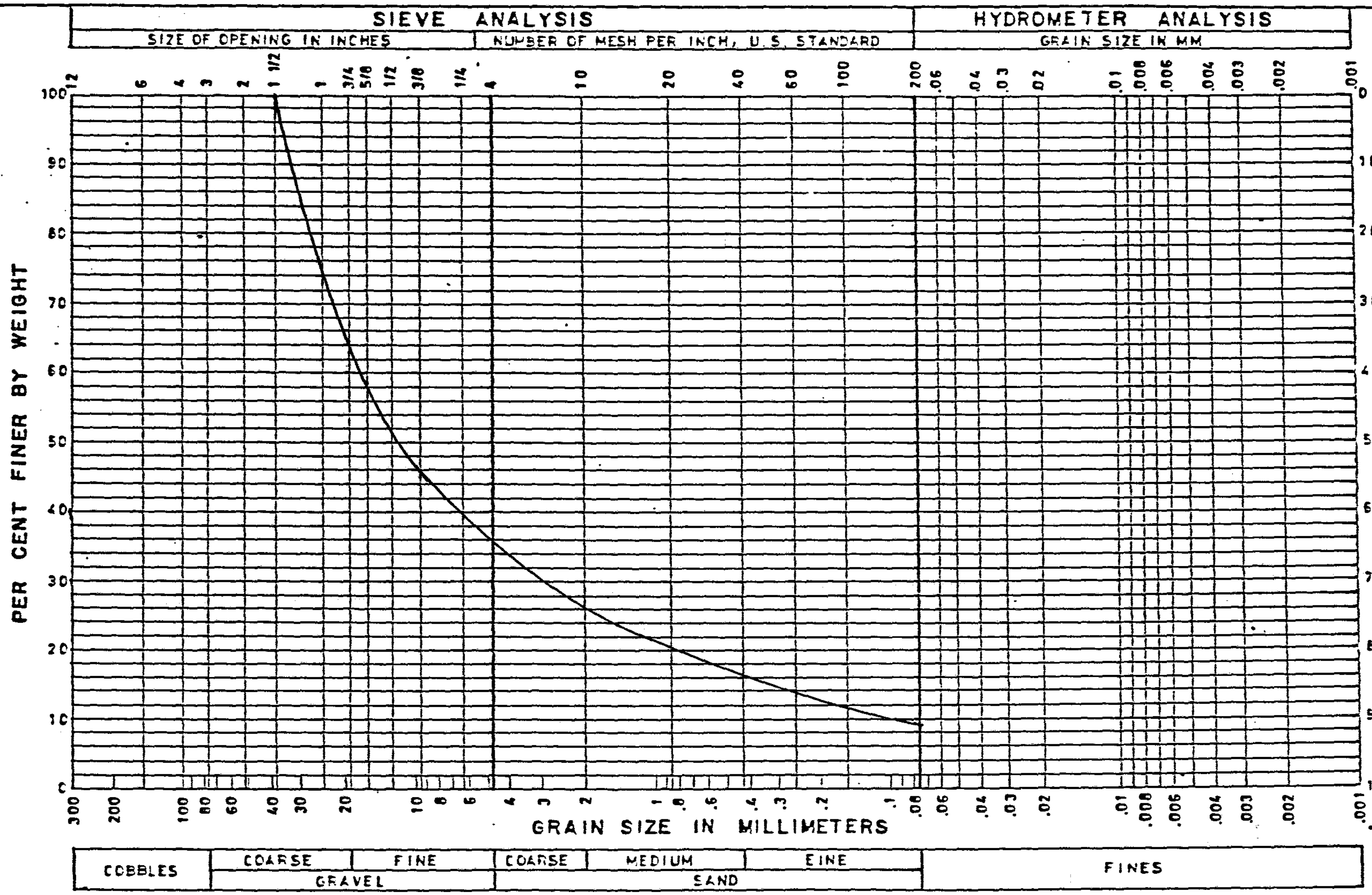

SAMPLE NO.	DEPTH -FT.	U.S.C.	CLASSIFICATION	NAT W.C.%	LL	PL	PI
1-9	60.0	GW	Brown Sandy GRAVEL				

GRAIN SIZE CLASSIFICATION

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Structures

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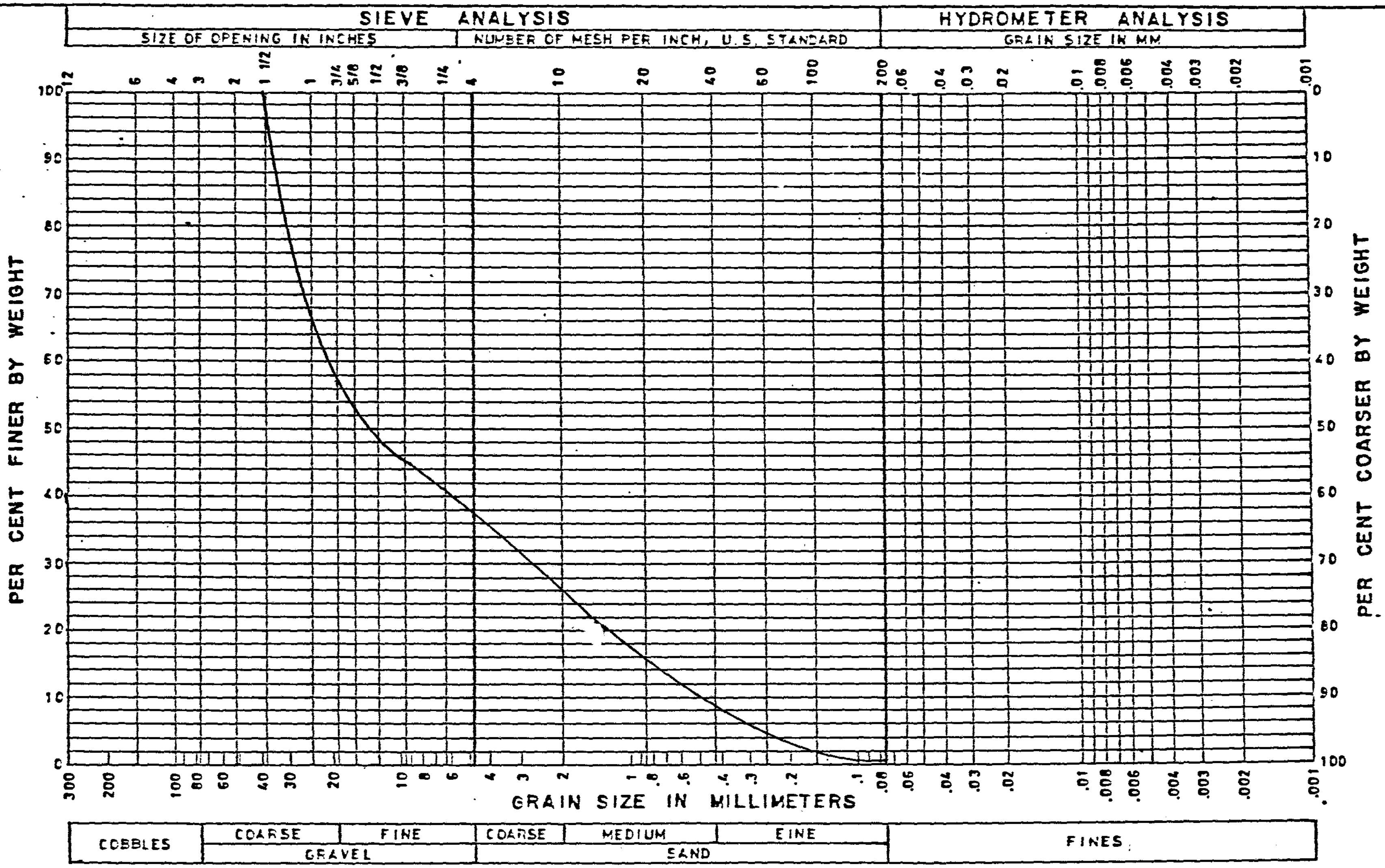

SAMPLE NO.	DEPTH -FT.	U.S.C.	CLASSIFICATION	NAT W.C.%	LL	PL	PI
1-3	15.0	GW	Brown Sandy GRAVEL				

GRAIN SIZE CLASSIFICATION

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SPT Co. Saratoga Creek Bridge
Structures

DATE: 9/89

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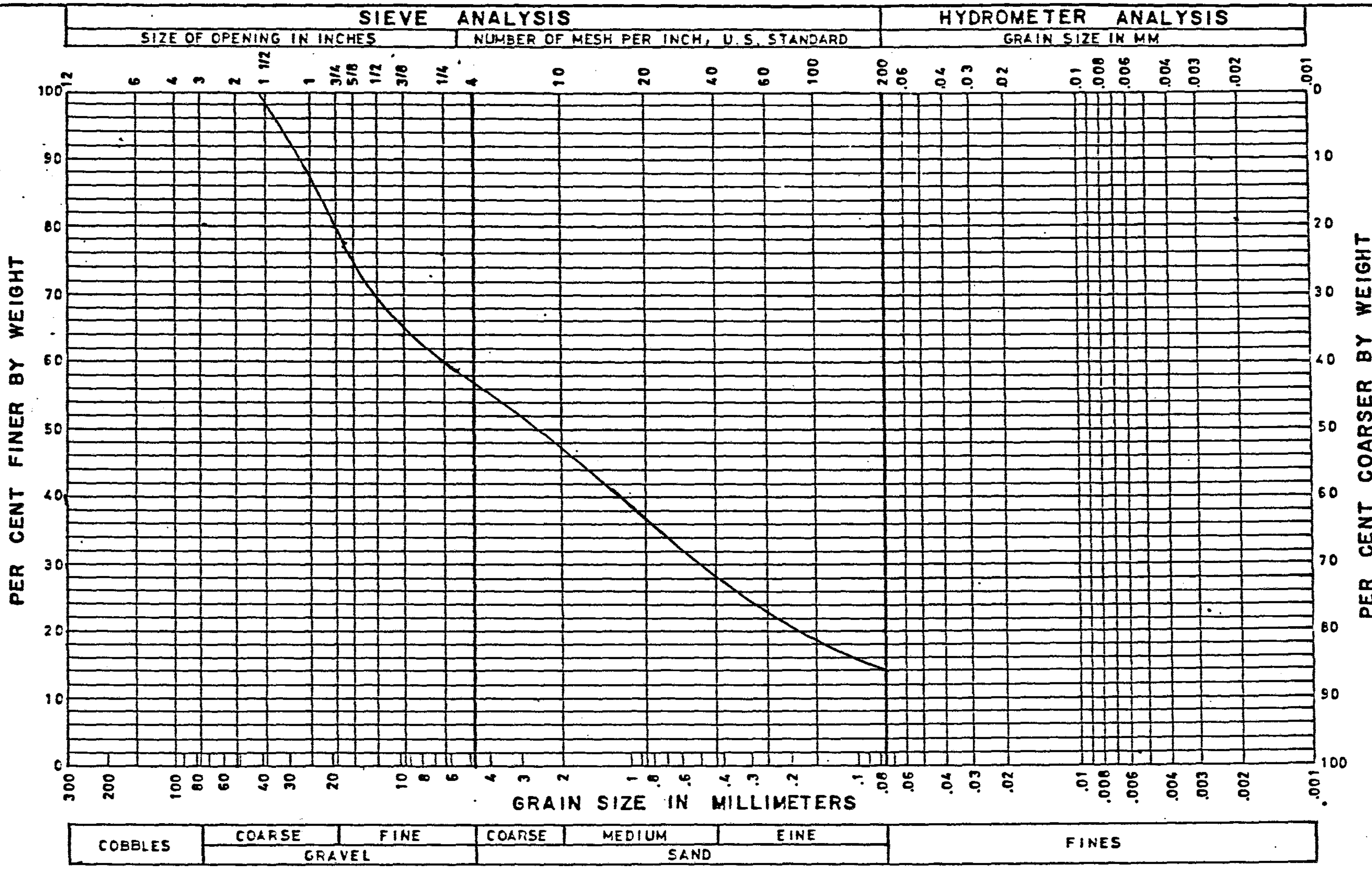

SAMPLE NO.	DEPTH -FT.	U.S.C.	CLASSIFICATION	NAT W.C.%	LL	PL	PI
2-6	50.0	GW	Brown Sandy GRAVEL				

GRAIN SIZE CLASSIFICATION

APPENDIX B

RECOMMENDED GENERAL GRADING SPECIFICATIONS

1.1 General Description

1.11 These specifications have been prepared for general grading and site development of this project. PSC Associates, Inc., hereinafter described as the Soil Engineer, should be consulted at least 4 days prior to any work connected with the existence of these specifications.

1.12 The work shall include all clearing and grubbing, preparation of land to be filled including benching and keying, filling of the land, spreading, compaction, density testing of the fill, and disposal of excess materials to complete the grading of areas in conformance with the lines, grades and slopes as shown on the grading plans.

1.13 In the event that any unusual conditions, not covered by these specifications, are encountered during grading operations, the Soil Engineer shall be immediately notified.

2.1 Clearing, Grubbing and Preparing Areas to be Filled

2.11 All timber, roots, brush, sod, abandoned buildings (surface and subsurface), debris and other deleterious materials shall be removed to approved disposal areas. This work includes dust control as required to alleviate any nuisance on or about the site. No burning shall be permitted in areas to be filled.

2.12 All loose soil, organic matter subject to decay, topsoil and spongy material shall be removed from the surface upon which the fill is to be placed; the exposed surface shall be free from ruts, hummocks or other uneven features which would tend to prevent uniform compaction. The subgrade shall be approved by the Soil Engineer prior to placing fill.

2.13 Where ground surface to receive fill is steeper than a gradient of 5 horizontal to 1 vertical (5:1), a fill stabilizing key shall be excavated into a subgrade that is competent and suitable for receiving fill after all loose soil, organic matter, topsoil, and spongy material have been removed in accordance with paragraph 2.12. The bottom of the key should be at least 12 feet wide and should slope into the hillside at a gradient of at least 2%.

2.14 The ground upon which the fill is to be placed shall be plowed or scarified to the recommended depth (usually 6 to 8 inches). Scarification of subgrade may be waived upon written approval of the Soil Engineer.

2.15 After the area to receive the fill has been cleared, approved, and scarified, it shall be disced or bladed until it is uniform and free from large clods, brought to the proper moisture content by adding water and mixing or aerating, and compacted to not less than 90% of maximum dry density as determined by ASTM Designation No. D 1557-78. Testing will be performed by the Soil Engineer.

2.16 Loose soils removed in accordance with paragraph 2.12, if free from deleterious materials, may be incorporated in compacted fill.

3.1 Materials

3.11 The materials for the engineered fill shall be approved by the Soil Engineer at least four days before commencement of grading operations. Any imported materials must be approved before being brought to the site. The materials used shall be free of solid lumps, clods and any deleterious substances. No rocks larger than six inches in dimension will be permitted in mass fills unless they are placed at the direction of the Soil Engineer.

3.12 The native soil, if free of organic or other deleterious material, may be used as fill.

3.13 The contractor shall notify the Soil Engineer at least four working days in advance of his intention to import soil from any source outside the project areas and shall permit the Soil Engineer to sample as necessary for the purpose of making tests to establish the qualities of these materials. The contractor shall be responsible for all cost incurred in sampling, testing, analyzing, and determining the applicability of the materials for use on the site. The work shall be performed by the Soil Engineer.

4.1 Placing, Spreading and Compacting Fill Material

4.11 The fill material shall be placed in horizontal layers which, when compacted, shall not exceed 6 inches in thickness. Each horizontal layer shall be spread evenly and shall be thoroughly mixed to minimize segregation and to provide uniformity of material in each layer.

4.12 When fill material includes oversized rock, they shall be placed according to the recommendations of the Soil Engineer in areas designated as suitable for rock disposal. No rocks larger than 6 inches will be permitted closer than 5 feet below the finished grade.

4.13 When the moisture content of the fill material is below that necessary to obtain the required density, water shall be added and thoroughly mixed into the soil. When the moisture content of the fill material is above the necessary to obtain the required density, the fill material shall be aerated by blading or other methods.

4.14 After each layer has been placed, it shall be thoroughly compacted to not less than 90% of maximum dry density as determined by ASTM Designation No. D 1557-78. Testing will be performed by the Soil Engineer.

4.15 Compaction of each layer shall be continuous over its entire area and continued out to the finished slope face.

5.1 Tests

5.11 The test used to define maximum densities of all compaction work shall be the ASTM Designation No. D 1557-78. All densities shall be expressed as a percentage of the maximum dry density obtained in the laboratory by the foregoing procedure.

5.12 Field density tests will be performed by the Soil Engineer. The location and number of tests will be determined by the Soil Engineer or applicable specifications.

5.13 Earthwork shall not be performed without approval by the Soil Engineer. Deviations from these specifications will not be allowed except by written variations signed by the Soil Engineer.

6.1 Seasonal Requirements

6.1 No fill shall be placed during unfavorable weather conditions as determined by the Soil Engineer. After interruption of work due to heavy rain, the Soil Engineer shall approve previously placed fill before resumption of earthmoving operations.